Consideration on landslide mechanism based on pore water pressure loading test

Examan du mécanisme de glissement de terrains par essai de suppression hydrostatique

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ABSTRACT: Pore water pressure loading test of clay was conducted under the constant deviator stress. From the test results, the yield and failure stresses were newly defined and the yield stress was found located on the critical state line. With the increase in pore water pressure over the yield stress, the shear deformation developed with long time. The progressive failure with water migration was pointed out as the reason for the slow movement of landslide. Numerical simulation on shear strength of clay against repetitive pore water pressure loading clarified the process of deterioration in shear strength of clay caused by repeats of landslide movement.

RESUME: L'essai du chargement de la pression de l'eau interstitielle de argile a été procédé sous la contrainte constante de cisaillement. Selon les résultats, la contrainte d'écoulement et celle de rupture ont été définies et la première a été trouvée aux environs de la ligne de l'état critique. En augmentant la pression de l'eau interstitielle jusqu'au-dessus de la contrainte d'écoulement, la déformation de cisaillement s'est formée et s'est développée en longues heures. Le mouvement lent du glissement du sol était dû à la rupture progressive entraînée par la migration de l'eau. La simulation numérique sur la résistance au cisaillement de l'argile, chargée à plusieurs reprises la pression de l'eau interstitielle, a éclairci le processus de la diminution de la résistance au cisaillement de l'argile causée par la répétition des mouvements du glissement du sol.

1 INTRODUCTION

The feature of landslides in Hokuriku district in Japan is a repetitive failure of gentle slopes. The slip line has been formed in slope and the landslide takes place along it repeatedly. It is important to clarify the behavior of clay on the slip line to make clear the failure mechanism of those landslides.

Landslide has been explained by the reduction in effective stress due to the increase in pore water pressure by rainfall and melted snow. However, the reasons why the landslide takes place slowly contrary to the quick failure of slopes by rainstorm and repeats for long time, and how much the shear strength of clay is after the repeats of landslide movement have not been still clear.

This paper approached the landslide mechanism from three different ways as (1)pore water pressure loading test of clay, (2)simulation on shear strength change of clay due to repetitive pore water pressure loading and (3)physical property of clay on the slip line in landslide slope.

2 PORE WATER PRESSURE LOADING TEST

2.1 *Testing method*

The soil employed for the test was the clay sampled in Nagaoka city. The liquid and plastic limits of the soil were w_l=49.8% and w_p=35.7%, respectively. The soil was sifted with a 425μm sieve and preconsolidated at 47kPa. The soil specimen was set on the triaxial compression test apparatus and consolidated at 200kPa.

The soil specimen was first loaded with a certain deviator stress q_i under the undrained condition. Keeping the deviator stress as constant, the pore water pressure was enforced to increase at the bottom of specimen. The water pressure was measured at the top of specimen. The pore water pressure was increased step by step after confirming the enforced water pressure measured at the top of specimen. The increase in pore water pressure in a sep was set as 10kPa.

The deviator stress expresses the shear stress of slope caused by the slope inclination. In the test the deviator stress was changed widely to investigate the soil behavior for the pore water pressure increase.

2.2 *Pore water pressure behavior*

Figure 1 shows the measured pore water pressure in time at the top of soil specimen for the deviator stresses, 50 and 100kPa. The measured pore water pressure formed a convex relationship with time. It is because the pore water pressure was loaded step by step. The certain finite time is required for the pore water transmission from the bottom to the top of specimen. The transmission time seems to be almost same when the pore water pressure is low. However, it is readily seen to get longer with the increase in pore water pressure over the certain magnitude. At the last stage of pore water pressure loading the transmission time was very long and the soil specimen finally attained to failure. From the comparison between two different deviator stresses, it is readily seen that the magnitude in pore water pressure at failure was greater and the transmission time in the last stage was longer in the case of lower deviator stress of 50kPa. This fact suggests the deformation of soil specimen against the pore water pressure loading is much affected by the magnitude of applied deviator stress. It is noted that the pore water pressure transmission time gets longer over the certain magnitude in pore water pressure.

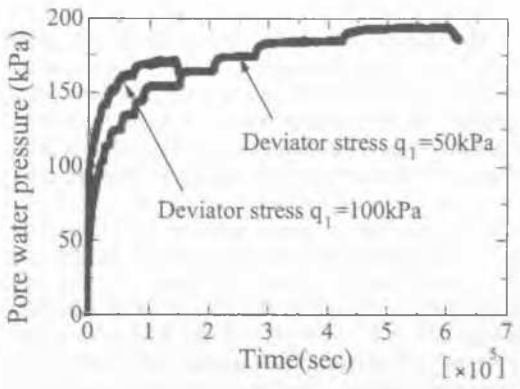

Figure 1. Measured pore water pressure in time at top of soil specimen.

Figure 2 expresses the axial strain and mean effective stress relationship in pore water pressure loading test of q_i=75kPa. The figure shows the relationship from the initial state of isotropic consolidation. The deviator stress was firstly applied under the undrained condition and the pore water pressure loading started from about p'=160kPa. The generation of axial strain was small in the former stage of pore water pressure loading, but increased at about p'=50kPa drastically. Ogawa (1986) pointed out the existence of threshold magnitudes in effective mean stress for deformation generation based on the ring shear test changing the constraint stress under the constant shear stress condition. He named the threshold magnitudes as the lower and upper yield limits which correspond to the yield and failure magnitudes, respectively.

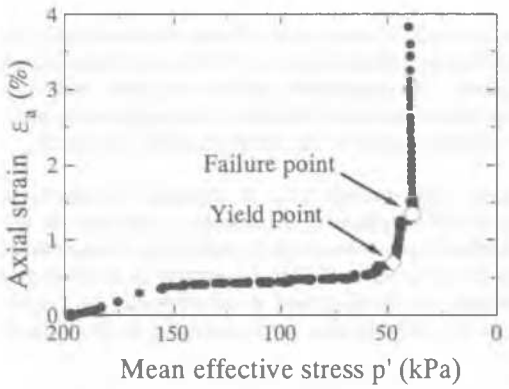

Figure 2. Mean effective stress versus axial strain relationship (q_i=75kPa).

2.3 *Effective stress and void ratio paths*

Figure 3 shows the effective stress and the void ratio paths for the case of deviator stress q_i=75kPa. The pore water pressure loading test started from the point b in the figure. With the increase in pore water pressure the effective stress moved leftward on the constant deviator stress line and the void ratio traced the swelling line. In the figure the void ratio increased largely from the point c. The increase in void ratio from the point c indicated the plastic deformation generated. This paper defined the point c as the yield point based on the void ratio and mean effective stress relationship. In pore water pressure loading test the soil is enforced both overconsolidated and sheared. The positive dilatancy property of overconsolidated clay caused the increase in void ratio as shown in the figure. With the further increase in pore water pressure the deviator stress could not keep constant at the point d. After the point d, the deviator stress dropped and the soil specimen rapidly failed. The failure point was defined as the point d based on the deviator stress and mean effective stress relationship in this study. The defined failure point corresponds to the peak strength of overconsolidated clay in the fully drained test.

In figure 3 the yield function of the original Cam clay model is drawn for pore water pressure loading. It can be seen that the plastic deformation generated even though the effective stress was in the elastic domain defined by the original Cam clay model. This tendency was commonly observed in other tests. It is difficult to present the soil behavior against pore water pressure loading with the use of the original Cam clay model.

Figure 4 shows the effective stress paths for the various deviator stresses, q_i. The critical state line of the clay is exhibited in the figure. It is readily seen that the yield points for the pore pressure loading located almost on the critical state line. The failure points were found to locate leftward the critical state line. The failure line was obtained as a non-linear curve contrary to the yield points. The effective stress seems to move toward the critical state line passing by the failure points. It is interesting that both the yield and the residual points almost located on the critical state line independent of deviator stress. It is, therefore, important to catch the failure and critical state lines in design practice.

The test was conducted for the case of q_i=25kPa, but the failure point could be obtained. It is because the test was quitted when the prescribed time was over. As shown in figure 1, the failure time gets longer with lower shear stress. It is supposed to fail in long time if the effective stress attains to the failure point predicted from the failure line in Figure 4.

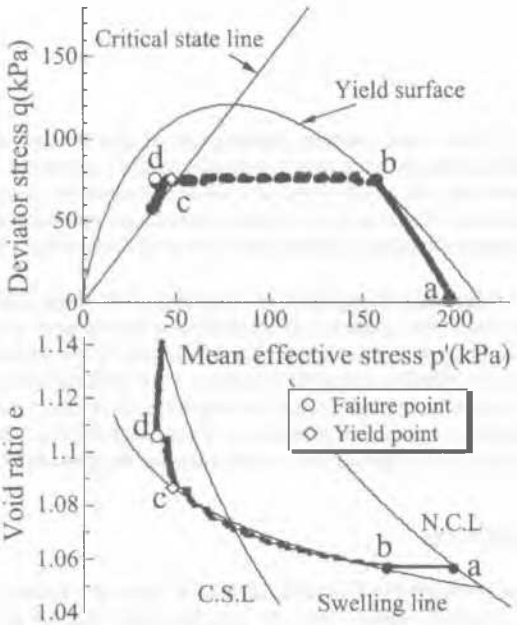

Figure 3. Effective stress and void ratio paths (q_i=75kPa).

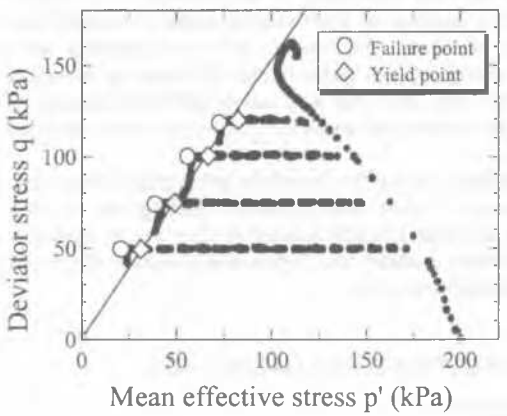

Figure 4. Yield and failure points in pore water pressure loading tests.

2.4 *Deformation and water migration*

Figure 5 expresses the failure mode and the void ratio distribution of soil specimen at failure. In the figure the apparent slip line formed in the specimen and the void ratio was widely distributed. This figure indicates the soil specimen lost the element property of homogeneity in stress and strain with shear deformation. The element property was lost around the failure point by the observation on specimen deformation. It is noted to pay much attention to the arrangement of test result over the failure point.

From the detailed comparison between failure mode and void ratio distribution, it can be seen that the largely sheared soil expanded by water absorption. It is because the soil became overconsolidated and then dilated with shear deformation due to pore water pressure loading. It is readily seen that the void ratio of soil surrounding the slip line got higher in the soil specimen. In saturated soil the deformation proceeds with water migration. The inhomogeneity of void ratio requires water migration in soil specimen. The Darcy law generally governs the water migration and it takes the finite time for water to migrate in impermeable soil as saturated clay. Asaoka *et. al.*(1999) pointed out the delayed behavior of overconsolidated clay in time from the viewpoint of water migration.

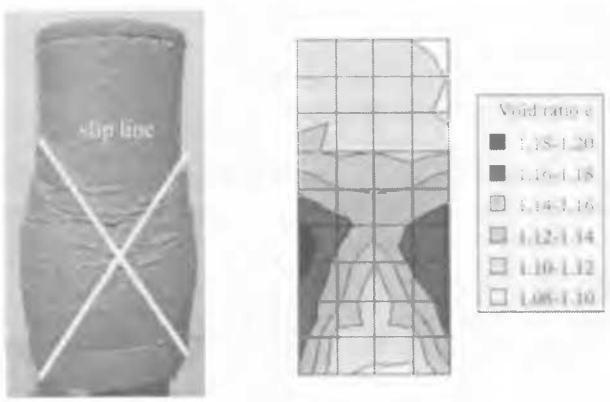

Figure 5. Soil specimen and void ratio at failure.

The water supply condition much affects the behavior of soil. The soil would behave as undrained without any supply of water, but behave as drained with enough water supplies. The shear strength of overconsolidated clay is higher under the undrained condition, but shows the strain softening under the drained condition. It means the shear strength of overconsoli-dated clay decreases with water absorption. It induces that the soil specimen might behave the progressive failure with the strain softening of overconsolidated clay induced by water absorption. The increase in pore water pressure transmission time in Figure 1 over the yield point is well explained by the progressive failure caused by strain softening of overconsoli-dated clay and water migration. The reason why the deviator stress decreased over the failure point in Figures 3 and 4 is that the shear resistance of soil specimen decreased by strain softening of overconsolidated clay and finally got smaller than the prescribed deviator stress.

3 SIMULATION BY CONSTITUTIVE EQUATION

The pore water pressure loading test was simulated with the use of constitutive equation. It is required for the constitutive equation to express the generation of plastic deformation shown in Figure 3 properly. The sub-loading surface model (Hashiguchi, 1989) was employed in this study. The yield function of the original Cam clay model was applied.

3.1 *Hashiguchi model*

Hashiguchi proposed the sub-loading surface that varies with the effective stress. It is expressed as follows:

$$f_s = q + Mp'\ln\frac{p'}{p_y'} - Mp'\ln R\,. \tag{1}$$

M denotes the critical state line parameter and R, the ratio of hardening parameter p_y' for the yield function and p_s' for the sub-loading surface as

$$R = \frac{p_s'}{p_y'}\,. \tag{2}$$

The plastic deformation is described by the behavior of the sub-loading surface, that is, the parameter R. The magnitude of the parameter R exists in the range from 0 to 1. The development of R obeys the following rule as Hashiguchi proposed.

$$\dot{R} = -\mu \ln R \left\| \dot{\varepsilon}^p \right\| \tag{3}$$

μ is the material constant. The clay behaves elastically when the sub-loading surface shrinks as $\dot{R} < 0$. On the contrary, it behaves elasto-plastically when the sub-loading surface expands as $\dot{R} > 0$.

3.2 *Simulation results*

Figure 6 shows the simulation of soil behavior for pore water pressure loading by the Hashiguchi model. It exhibits the relationship among shear and volumetric strains and mean effective stress. In the figure the volumetric strain was well simulated by the model, but the shear strain was not so much. However, the yield and failure points were almost predicted. The Hashiguchi model well described the shear deformation of overconsolidated clay taking account of dilation property. Table 1 indicates the employed parameter in the model.

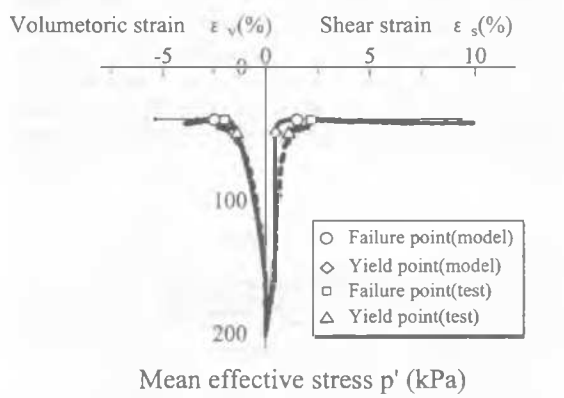

Figure 6. Numerical simulation of shear and volumetric strains.

Table 1. Soil parameters.

M	1.53	ν	0.2
μ	20	e_o	1.057
λ	0.111	κ	0.025

Figure 7 expresses the yield and the failure points of overconsolidated clays for pore water pressure loading. The overconsolidation ratio was widely varied in the figure. It is possible to compare the predicted values for OCR=1 with the experiment in Figure 4. From the comparison between two, the yield and the failure points were well simulated by the model for various deviator stresses. It is readily seen that the yield points was on the critical state line for various OCRs, but the failure points varied more leftward to the critical state line. It means the heavily overconsolidated clay could stand the higher pore water pressure loading.

3.3 *Repetitive pore water pressure loading*

Figure 8 indicates the computed result on the relationship between void ratio and mean effective stress for repetitive pore water pressure loading. In the figure the heavily overconsolidated clay was loaded repeatedly under constant deviator stress of q_i=75kPa. The void ratio of clay greatly increased at the fail-

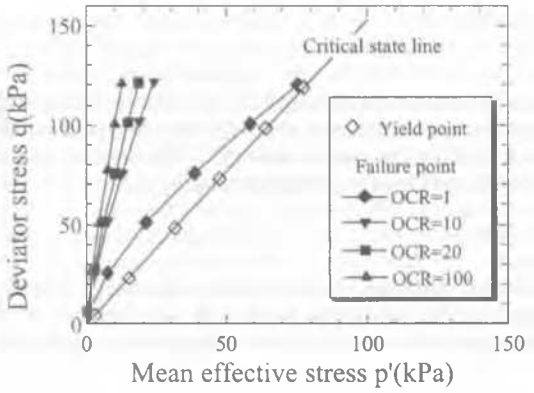

Figure 7. Computed failure points for various OCR.

ure point and monotonically increased by repetitive failure. It means the overconsolidated clay was getting to be close to normally consolidated state. It suggests the steady state of soil exists for repetitive loading.

The figure shows the failure line for the specific deviator stress q_i in the void ratio and mean effective stress space. The failure line was given by the curve and it expresses that the soil will fail due to smaller magnitude in pore water pressure by the experience of repetitive failure. In the figure three failure lines were exhibited under deviator stresses of q_i=50, 75 and 100kPa. These shows the soil could stand the higher pore water pressure for the lower deviator stress. This figure schematically illustrates the change of shear strength of landslide slope where the deviator stress denotes the failure potential of slope. The shift in failure line expresses the process that the slope stabilizes by repetitive landslides.

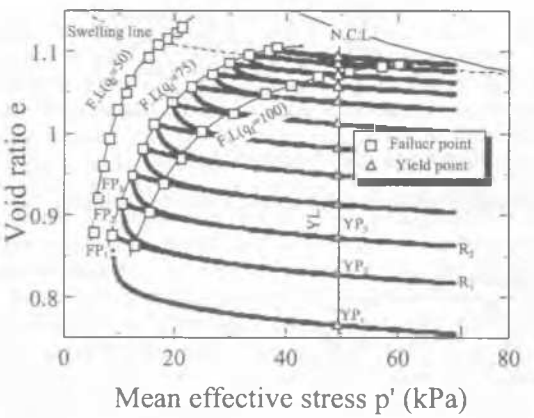

Figure 8. Failure lines in void ratio and mean effective stress apace.

4 PHYSICAL PROPERTY OF LANDSLIDE CLAY

The yield and residual points in test were related with the critical state line, that is, the angle of shear resistance ϕ'. It is, however, noted that gentle slopes still caused landslides in Hokuriku district in Japan. Even though the possibility of high pore water pressure inside the slope, it is difficult to explain the landslide without introducing the lower angle of shear resistance. The inherent mechanical property of landslide clay has been investigated vigorously. The effect of mineral constitution of soil on its mechanical property is noted as the factor of lower angle of shear resistance.

Figure 9 shows the relationship between the angle of shear resistance and plasticity index (JGS, 1988). The angle of shear resistance decreases with the increase in plasticity index. Table 2 denotes the physical test results on sampled soil from cites. The plasticity index of clay sampled from the slip line in Tochio landslide was obtained very high and it means the angle of shear resistance might be small based on the data in figure 9.

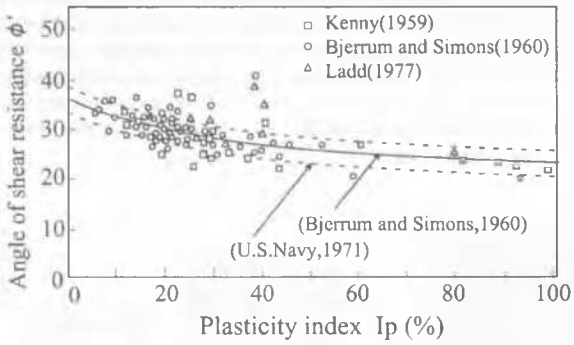

Figure 9. Relationship between angle of shear resistance and plasticity index.

Table 2. Physical test on sampled clay.

clay	type	ω (%)	Ip (%)
Tochio	slip line	154	347
	beneath slip line (with fine rock)	80.4	233
Nagaoka		40.1	14.1

5 CONCLUSIONS

The followings were concluded in this study.

1. Shear deformation of clay for pore water pressure loading took place for long time as the effective stress approached to the failure point. It was clarified that this behavior was well explained by the progressive failure of overconsolidated clay developing with water migration.
2. Pore water pressure loading tests for various deviator stresses gave the yield and the failure points. The yield points were located on the critical state line. The Hashiguchi model could predict both yield and failure points properly.
3. Change in failure point caused by repetitive pore water pressure loading was simulated with the Hashiguchi model. The process of heavily overconsolidated clay approaching to normally consolidation was clarified.
4. The plasticity index of clay sampled from landslide cite was obtained very high and it was one of reasons why gentle slopes still landslided in Hokuriku district in Japan.

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